Available online at www.eccomasproceedia.org
Eccomas Proceedia COMPDYN (2023) 2725-2740

ECCOMAS
Proceedia

COMPDYN 2023
9th ECCOMAS Thematic Conference on
Computational Methods in Structural Dynamics and Earthquake Engineering
M. Papadrakakis, M. Fragiadakis (eds.)
Athens, Greece, 12-14 June 2023

MULTI-RISK ASSESSMENT OF A BASE ISOLATED STRUCTURE

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Abstract

Seismic isolation is a design strategy adopted for the protection of both new and existing structures, based on the assumption that it is possible to uncouple a structure from the ground and thereby protect it from the damaging effects of earthquakes. The most common isolators are the elastomeric ones, which consist of alternating rubber layers and reinforcing metal sheets, connected by vulcanization of the elastomeric compound, and a lead central core. In case of a fire, this compound can suffer permanent damage which can seriously compromise the functionality of the isolation system. This study is focused on the assessment of the fire effect on the elastomeric isolators and the quantification of their loss of efficiency after a fire event.

At this purpose, a case study of an existing r.c. structure with multi-storey frames was analysed, for which, after a seismic vulnerability assessment, two possible interventions with seismic isolation were designed. The evaluation of the isolator thermal response for 30 and 60 minutes of exposure to the ISO 834 standard fire curve was used for quantify the damage induced by the fire in terms of degradation of vertical and horizontal stiffness and vertical load-bearing of the isolators. This damage level was defined starting from the thermomechanical properties of the elastomeric layer damaged by fire which, in particular, reached temperatures greater than a critical temperature threshold (70, 200 and 320°C). The estimated degradation was used to evaluate the seismic performance of both isolators and structure in different design configurations, also in a multi-risk scenario that foresee the occurrence of a seismic event after a fire. Finally, in order to mitigate the fire effect on the elastomeric isolators, the effectiveness of a passive fire protection system consisting of plasterboard gypsum boxes was evaluated, ensuring that the critical temperature of 70°C on the outer surface of the isolator was not exceeded even after 60 minutes of exposure to standard fire.

Keywords: Fire Risk, Seismic Risk, fire after earthquake, isolated r.c. structures

ISSN:2623-3347 © 2023 The Authors. Published by Eccomas Proceedia.
Peer-review under responsibility of the organizing committee of COMPDYN 2023.
doi: 10.7712/120123.10596.20705

1 INTRODUCTION

Earthquakes are among the most dangerous natural disasters, with loss of human lives, a large number of structural damages and socio-economic costs. As it is not possible to prevent earthquakes from occurring, designers have to focus on mitigating their consequences. This goal can be achieved by both increasing the structural capacity and ductility or decreasing the seismic demand of the structure. This latter solution can be reached by using the base isolation, reducing seismic induced damage. Indeed, the design of seismic isolation is a topic of great interest and it is considered a consolidated technique in the field of seismic protection of structures. However, during the nominal life of the structure, it is important to also consider the possibility that an extreme event like fire, as consequence of a main seismic event, can occur, compromising the seismic isolation system functionality and making it more vulnerable to the seismic action of an after-shock. Nowadays, the more widely used seismic isolators are the steel reinforced elastomeric ones, that are devices composed by alternate layers of elastomer and steel, connected through the vulcanization of the elastomeric compound [1]. Because of the high temperatures that can be reached during a fire event, the elastomer can suffer not reversible damages, with a decay of the mechanical properties of isolators.

In scientific literature, only few studies concerning the evaluation of fire induced damage of seismic isolators and the estimation of their mechanical properties degradation are available. In [2], the authors deal with the analysis of the thermo-mechanical behaviour of polychloroprene elastomeric isolators during a fire, proponing a model able to reproduce the behaviour of the elastomer. Based on test results, the authors set at the temperature of 320°C the onset of degradation with a significant decrease of density and, consequently, by the loss of structural integrity of the material. The model is validated comparing the results of an experimental test carried out on a specimen of elastomeric isolator with the results of a numerical simulation analysis. The study shows a good agreement between the results. In [3], the authors tested eight seismic elastomeric isolators, both in natural rubber (NRB) and with a lead core (LRB), with and without fire protection. The isolators are exposed to the standard fire curve ISO834 in order to evaluate the effects of the vertical load and fire duration on their collapse mechanisms and residual mechanical properties. In particular, the experimental results show that (i) the duration of fire exposure has an important effect on the decay of the mechanical properties of the isolators; (ii) the vertical load does not affect the residual mechanical properties and the fire resistance; (iii) the proposed passive protection system prevents an important decay of the mechanical properties of elastomeric isolators in case of fire. After 30 min of fire exposure, the mechanical properties of LRB and NRB isolators have a decay lower than 10%, while after 60 min the vertical stiffness of LRB isolators and their mechanical properties dependent by the lead core (yielding stress and cyclic energy dissipation) do not have significant variations. A more significant variation can be observed on the mechanical properties dependent by the rubber. A decrease of about 36% of the post-elastic stiffness, an increase of 20% of the equivalent damping ratio and a decrease of about 20% of the equivalent lateral stiffness are observed. For NRB isolators, the vertical and lateral stiffness decrease of 27% and 62%, respectively. With fire protection, the seismic isolators do not suffer significant changes, even after 3 hours of fire exposure, with a reduction of the mechanical properties lower than 8%. In [4], the authors evaluate the degradation of the mechanical properties of HDLRB (High-Damping-Laminated-Rubber Bearings) and LRB (Lead-Rubber Bearings) isolators based on the temperature distribution inside them, obtained with numerical analyses. As in the method of isotherm 500°C, proposed by EN1992-1-2 [5] for the evaluation of the residual properties of reinforced concrete cross sections damaged by fire, the authors propose the method of isotherm 200°C, to evaluate the reduction of the mechanical properties of the isolators damaged by fire. In this method, the residual cross section of HDLRB and LRB isolators is obtained, neglecting the elastomer with temperatures higher than the vulcanization threshold.

All the studies proposed in literature are mainly focused on the behavior of the single isolator, neglecting the behavior of the whole base-isolated structure. Furthermore, the topic of the residual seismic capacity of isolators after a fire event is very important, but a lack of knowledge and uncertainties exist. Therefore, the paper aims at evaluating the behavior of elastomeric isolators before and after a fire event and the consequences that their damage can have on the whole structural behavior.

2 EVALUATION OF SEISMIC AND FIRE ACTIONS

In this section, the methodology used to evaluate the seismic risk occurring after a fire event of existing seismically isolated buildings was introduced. In particular, (i) the methodology used for the evaluation of the seismic vulnerability of the structure, (ii) the criteria used for the design of the seismic isolation system and (iii) the methodology used to take into account the fire risk is shown.

2.1 Design of the seismic isolation system

To design the seismic isolation system for an existing structure, the first step is the assessment of its seismic vulnerability. In this paper, the evaluation of the seismic vulnerability is performed through response spectrum linear dynamic analyses. The response spectrum used is related to a life-safety limit state (SLV according to Italian Technical Code for Constructions - NTC2018 [6],[7]), with reference to the specific site conditions, reduced by a behavior factor q=1.5 (cautionary value, typical for existing structures designed for vertical loads only). NTC2018 provides response spectra and a seismic design approach very similar to EN1998 [8], but considering the specific seismic hazard of Italy. The results of the seismic vulnerability assessment are necessary for the design of seismic strengthening interventions, that, in general, could include: a) reinforced concrete jacketing of beams and columns under the seismic isolation interface, b) installation of elastomeric seismic isolators and sliding supports (multi-directional structural bearings with confined elastomeric disc) on the top of the columns, under the isolation interface. The installation of seismic isolators and sliding supports can be realized following the operative phases shown in [9]. The seismic isolation system is designed considering the elastic response spectrum corresponding to the Collapse Prevention Limit State with a viscous equivalent damping of 15% for periods (T) greater than 0.8 T_{iso}, with Tiso equal to the fundamental vibration period of the seismically isolated structure [6]. So, knowing the total mass of the structure M_{tot} and assuming a “target” design period of the seismically isolated structure (defined on basis of the seismic vulnerability assessment and on the desired reduction of spectral acceleration), from the equation:

$$T_{iso} = 2\pi\sqrt{\frac{M_{tot}}{K_{tot,iso}}} \tag{1}$$

it is possible to calculate the “target” stiffness of the whole isolation system, as:

$$K_{tot,iso} = \frac{4\pi^2 M_{tot}}{T_{iso}^2} \tag{2}$$

and the “target” stiffness related to the single isolator, as:

$$K_{iso,i} = \frac{K_{tot,iso}}{n_{iso}} \tag{3}$$

in which n_{iso} is the number of isolators.

The layout in the plan of seismic isolators and sliding supports is defined trying to minimize as much as possible the eccentricity between the effective stiffness and mass centers at the isolation interface (e.g. 3% of the plan dimensions of the building in the two directions [6]). Once the isolation system is designed, a further evaluation of the structural seismic performances is performed, to check the safety of the seismic isolators in terms of loads, stresses, displacements, and deformability (see [7]).

2.2 Assessment of fire damage

To quantify the fire damage of seismic isolators, the degradation of mechanical properties, such as horizontal stiffness K_H, vertical stiffness K_V and load-bearing capacity have to be evaluated. This evaluation can be conducted using a methodology similar to the one proposed in the EN1992-1-2 [5], for verifying the fire resistance of reinforced concrete elements, named "500°C isotherm method". In particular, the methodology considers a reduced cross-section in the calculation of load bearing capacity, neglecting all the concrete that has reached temperatures greater than 500ºC. The remaining concrete cross-section maintains its initial strength and stiffness values. Imagining to "adapt" this approach to elastomeric seismic isolators, the first problem is to establish a unique critical temperature threshold, since few and different information are provided in scientific literature. This temperature seems to be related to chemical phenomena that occur inside the elastomeric compound when it is exposed to elevated temperatures, so it could be obtained only by experimental tests. At this purpose Traxl et al. [2] stated that the temperature of 320 °C can be considered as an upper bound for the estimation of the residual material stiffness, since, above that temperature, the degradation process starts, with a significant decrease of density and integrity of the material. Mazza [4] suggested a more conservative temperature threshold of 200 °C, equal to the temperature of vulcanization between rubber layers and steel plates. The rubber aging behavior must be investigated by performing accelerated aging, according to ISO 18[9] , conditioning the sample at 70°C for a certain period linked to the compound type. Therefore also 70°C was chosen as temperature threshold value[10]. In conclusion, in this study, the assessment of fire damage of seismic elastomeric isolators is conducted with reference to critical temperature thresholds of 70, 200 and 320 ºC. The behaviour of elastomeric isolators is characterized by two main geometric parameters, defined as following (see Figure 1):

- $S_1 = A'/L = D'/4t_i$, called primary shape factor, defined as the ratio between the surface A' in common between the single layer of the elastomer and the steel sheet, and the free lateral surface L of the single layer of elastomer;
- $S_2 = D'/t_e$, called secondary shape factor, defined as the ratio between the plan dimension D' of the single steel plate and the total thickness of the elastomer layers t_e.

The first one (S_1) affects the vertical stiffness K_v of the isolator (controlling the rubber confinement), while the second one (S_2) is related to the possible occurrence of instability phenomena. For the calculation of K_v, only the area of the steel plate layer is considered (A') because the outer lateral layer of unconfined elastomer provides a negligible contribution to the bearing capacity against vertical loads.

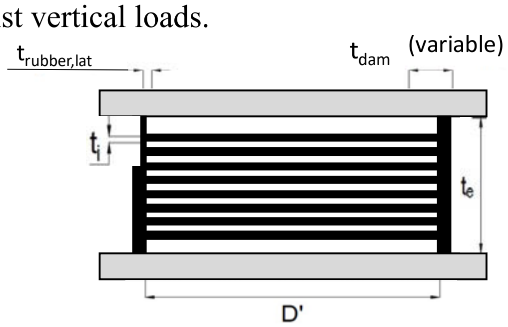


Figure 1. – Geometry of isolators.

Given the vertical stiffness of the elastomer single layer:

$$K_{Vi} = \frac{E_c * A\prime}{t_i} \tag{4}$$

the vertical stiffness K_v of the isolator can be calculated considering each single layer in series, as:

$$K_V = \frac{1}{\sum \frac{1}{K_{vi}}} \tag{5}$$

The axial compressibility modulus E_c depends on the type of elastomeric compound (dynamic shear modulus of the elastomer G_{din}) and the primary shape factor S_1, and it can be evaluated [7]:

$$E_c = \left(\frac{1}{6 * G_{din} * S_1^2} + \frac{4}{3 * E_b}\right)^{-1} \tag{6}$$

where E_b is the volumetric modulus of the rubber, which can be assumed equal to 2000 MPa if a direct measure is not available [7].
The horizontal stiffness of the isolator can be obtained from:

$$K_H = \frac{G_{din} * A}{t_e} \tag{7}$$

where G_{din}, A and t_e are the equivalent dynamic shear modulus, the gross cross-section and the total thickness of the elastomer, respectively. All these illustrated aspects were considered in this work for evaluation of the isolator mechanical properties degradation (horizontal K_H and vertical K_V stiffnesses) induced by the fire. In particular, this degradation depends on two main parameters such as the critical temperature threshold θ_{crit} and the time of exposure to fire, t. In the equation (7) there is a direct dependence of the stiffness K_H on the cross-section A and so on the diameter D. Therefore, assuming that the mechanical properties of the rubber (G_{din}, E_b) and the geometric characteristics of the isolator (t_e, t_i) do not change due to fire, an effective area reduced by the fire can be evaluated on the basis of thermal damage.

For the residual horizontal stiffness calculation, a diameter Dfinal,i was considered:

$$D_{final,i}(\theta_{crit}, t) = D - 2 * t_{dam,i} \tag{8}$$

In (8) $t_{dam,i}$ is the fire damaged isolator portion, variable with the thermal field induced by fire in the isolator, corresponding to the external portion of isolator with temeperature greater the fixed threshold ($\theta > \theta$ crit) . For the calculation of the residual vertical stiffness, a diameter D'final,i was considered:

$$\begin{aligned} D'_{final,i}(\theta_{crit}, t) &= D' - 2 * (t_{dam,i} - t_{rubber\ lat.}) \qquad && if\ t_{dam} \geq t_{rubber\ lat.} \\ D'_{final,i}(\theta_{crit}, t) &= D' && if\ t_{dam} < t_{rubber\ lat.} \end{aligned} \tag{9}$$

where $t_{dam}(\theta_{crit}, t)$ is the damaged portion and trubber lat represents the rubber lateral layer which has a thickness of 10 mm and is inserted for protecting the layer of elastomer and steel plates (see Figure 1). The second equation of (9) is verified if the damaged portion t_{dam} is less than the outer lateral layer of unconfined elastomer ($t_{rubber\ lat}$), which, as also said before, is not considered for the vertical stiffness calculation. So, in this case the vertical stiffness is not affected by fire damage. Since the value D'_{final} affect the calculation of S_1 and E_c , in case of fire, each layer "i" has a different value of S_1 and E_c, according to the damage state of the layer. Therefore, the post-fire stiffness K_H (θ_{crit}, t) can be obtained from the equation (10), replacing "A" with "$A_{effective}$", obtained as the mean of the effective area of each elastomeric layer Aeffective,i with a diameter D_{final}. While A' has to be replaced with $A'_{effective,i}$ in the equation (7) for calculating the vertical post-fire stiffness of the single layer K_{Vi} (θ_{crit}, t). Finally, the equations of the reduction factors for horizontal (α_{kh}) and vertical (α_{kv}) stiffnesses, for the three critical temperature thresholds, can be expressed as following:

$$\alpha_{kh}(\theta_{crit}, t) = \frac{K_H(\theta_{crit}, t)}{K_{H,0}} \tag{10}$$

$$\alpha_{kv}(\theta_{crit}, t) = \frac{K_V(\theta_{crit}, t)}{K_{V,0}} \tag{11}$$

where $K_H(\theta_{crit,\ t})$ and $K_V(\theta_{crit,\ t})$ are the post-fire stiffnesses obtained for a certain critical temperature threshold at the time t of fire exposure, while $K_{H,0}$ and $K_{V,0}$ are the initial stiffnesses at t=0 min. Considering the load-bearing capacity of the isolators F_{zd} at the ULS, the residual load-bearing capacity $F_{zd}(\theta_{crit},t)$ due to the effect of fire can be estimated as:

$$F_{zd}(\theta_{crit}, t) = F_{zd} * a_{eff}(\theta_{crit}, t) \tag{12}$$

where $a_{eff}(\theta_{crit}, t)$ is the effective area obtained as:

$$a_{eff}(\theta_{crit}, \boldsymbol{t}) = \frac{A_{eff}(\theta_{crit}, t)}{A} \tag{13}$$

To use the reduction factors of the ULS conditions is a precautionary choice, indeed in case of fire these factors are considered equal to one [9].

3 APPLICATION TO A CASE STUDY

The methodology shown in the previous sections was applied to a reinforced concrete existing building. In particular, the results of the seismic vulnerability assessment and the seismic strengthening intervention are described in detail. Moreover, the effect of the fire on the seismic isolation intervention is considered, proposing possible design measures that could be used in practice to consider seismic and fire risk in a combined approach.

3.1 Case study description

The case study refers to an existing building made of reinforced concrete frames. The structure, built in 50s, has one underground floor and four floors above ground (total height of 19.80 m) and has a rectangular shape plan with dimensions 30 m x 12 m.

The building, designed for vertical loads only and without seismic requirements, has main frames in the direction of the longer side of the plan; the slabs, made in reinforced brick concrete, are mainly oriented in the direction of the shorter side of the building. There are not connecting beams between the main frames, except for the basement floor. In Figure 2, a typical floor carpentry of the building is represented.

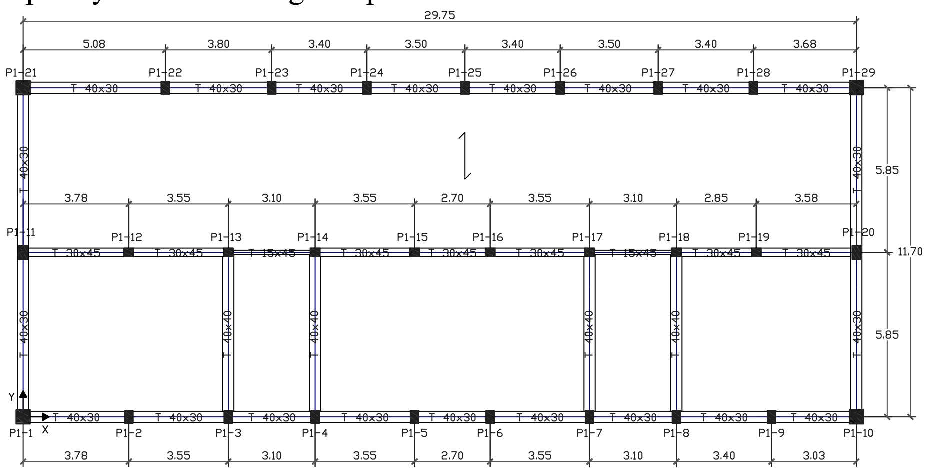


Figure 2 – Typical floor with numbering of columns and dimension of beams

The dimensions of the beams cross section are the following:

- 50x40 cm, 40x40 cm and 25x40 cm for the basement floor;
- 45x30 cm, 40x40 cm and 20x50 cm for the first floor;

- 40x30 cm, 40x40 cm, 30x50 cm and 15x45 cm for intermediate floors;
- 40x45 cm, 40x40 cm, 30x40 cm and 15x35 cm at the last floor.

The dimensions of the columns cross sections are shown in Table 1 and each column is indicated as Px-y, where x indicates the floor number, while y the number of the column (see Figure 2). The mechanical properties of the materials were defined based on the results of some destructive and non-destructive tests performed on the structural members, which allowed defining the following mean resistances: f_{cm} = 27 MPa and f_{ym} = 337 MPa for concrete and rebar steel respectively.

3.2 Seismic vulnerability analysis for existing structure

A seismic vulnerability analysis of the case study building was performed, following the approach previously described in section 2.1. The numerical analyses were carried out with the software SAP2000 [12]. The structure was modelled through "frame" elements, while the infinitely rigid behaviour of the floors was modelled through the introduction of "diaphragms" constraints. Figure 3 shows a 3D view of the numerical model.

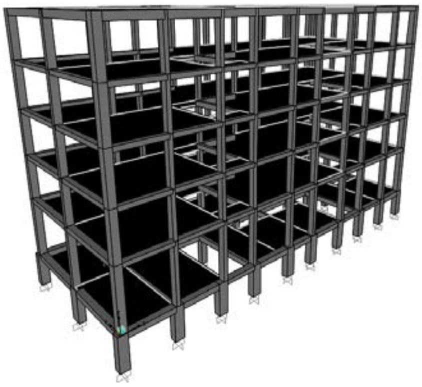


Figure 3 – Structural model of the building.

Table 1 – Cross sections of columns

Floor	Column	B [cm]	H [cm]
Basement	1,10,21,29	65	50
	2,3,4,5,6,7,8,9,22,23,24,25,26,27,28	70	50
	11,20	50	85
	12,13,14,15,16,17,18,19	50	50
Ground floor	1,10,21,29	50	50
	2,3,4,5,6,7,8,9,22,23,24,25,26,27,28	30	50
	11,20	30	50
	12,13,14,15,16,17,18,19	40	40
Floors 1,2,3	1,10,21,29	50	50
	2,3,4,5,6,7,8,9,22,23,24,25,26,27,28	30	45
	11,20	30	50
	12,13,14,15,16,17,18,19	35	30
Fourth floor	1,10,21,29	50	50
	2,3,4,5,6,7,8,9,22,23,24,25,26,27,28	30	45
	11,20	30	50
	12,13,14,15,16,17,18,19	30	30

Based on the internal forces obtained through the numerical analyses, the safety checks of structural elements were conducted. Table 2 shows the maximum values of the ratio Demand/Capacity (D/C) obtained for beams and columns. The design strength of the materials are defined on the basis of the average values and assuming a Full Knowledge Level (LC3 according to [6], confidence factor CF=1.0), also thanks to the availability of the original design drawings and the large number of in-situ inspections performed on the structure. For shear safety checks (brittle failures) the resistances of the materials were also divided by the partial safety coefficients (γ_c = 1.5 for concrete and γ_s = 1.15 for rebar steel).

Table 2 – Current state: safety checks of beams and columns

Check	Combination	$(D/C)_{max}$
Beams Bending	ULS	1.72
	Seismic	3.55
Beams Shear	ULS	0.74
	Seismic	0.94
Columns Combined Axial Load/Bending	ULS	0.93
	Seismic	3.14
Columns Shear	ULS	0.59
	Seismic	1.70

Based on the results of the seismic vulnerability evaluation of the structure in its current configuration, a seismic strengthening intervention, which consists of a seismic isolation system and the reinforced concrete jacketing of some beams and columns, was designed. More details on the intervention are shown in the following sections.

3.3 Seismic strengthening of the structure

The intervention of seismic isolation includes also the realization, under the isolation interface, of stiffening connecting beams with cross section 40x40 cm (second span in the short direction of the building, see Figure 2). These beams, that connect the columns of the central frame (columns 12, 13, 14, 16, 17, 18, 19) and those of one of the external frames (columns 22, 23, 24, 25, 26, 27 and 28), are intended to provide a greater stiffness of the isolation interface in its own plane. The reinforced concrete jacketing of the beams leads to the increasing of the section dimensions, in particular the sections 50x40 become 60x60, the sections 25x40 become 35x60 and the ones 40x40 increase up to 50x60. Furthermore, it provides the addition of 2-3 bars of diameter 16 or 20 mm, for both positive and negative bending. The reinforced concrete jacketing of the columns at the first level provides a thickness of 10 cm of concrete class C40/50 on all the sides of the existing cross section, in order to guarantee an adequate cover for new rebars (diameter 16 mm). Concerning the stirrups, rebars of 10 mm diameter spaced of 10 cm are provided, based on the prescriptions of Italian NTC2018 [6] for DC "B" (low ductility class). In particular, the existing columns of cross section 65x50 become 85x70, the ones 50x85 become 70x105, the ones 70x50 become 90x70 and, finally, those 50x50 become 70x70. The cross sections of the jacketed columns were modelled discerning the constitutive laws of existing concrete (f_{cm} = 27 MPa) and new one (f_{ck} = 40 MPa, f_{cd} = 23 MPa) as well as the constitutive laws of existing bars (f_{ym} = 337 MPa) and new ones (f_{yk} = 450 MPa, f_{yd} = 391 MPa). For concrete a parable-rectangle constitutive law was used (with ultimate deformation equal to 3.5‰), while for steel an elasto-plastic constitutive law was considered. The Figure 4 shows elastic response spectra at the Collapse Limit State (SLC according to [6]) in terms of acceleration and displacement. In the figure, the fundamental vibration periods of the structure with and without seismic isolation are also shown: T_1 refers to the fundamental period of the existing structure, while T_{iso} is the target period used for the design of the seismic isolation system. In Table 3, the values of spectral accelerations and displacements corresponding to the periods T_1 e T_{iso} are listed. The isolation system, designed to obtain an increase of the fundamental period from 1.49s to 3.0s, is able to decrease the elastic demand in terms of acceleration of 77% and increase that in terms of displacement of 4% (at the SLC).

Table 3 – Spectral accelerations and displacements at the SLC before and after seismic isolation

Existing structure		"Isolated" structure	
ξ	5 %	**ξ_{iso}**	15 %
T_1	1.49 s	**T_{iso}**	3.0 s
$S_{a,el}(T_1)$	0.13 g	**$S_{a,el}(T_{iso})$**	0.03 g
$S_{d,el}(T_1)$	7.41 cm	**$S_{d,el}(T_{iso})$**	7.74 cm

Assuming a target period of the isolated structure equal to 3.0s, given the total mass of the building M_{tot} = 1843.95 ton, the global stiffness of the isolation system was obtained:

$$K_{tot,iso} = \frac{4\pi^2 M_{tot}}{T_{iso}^2} = \frac{4\pi^2 1843.95}{3.0^2} = 8088\ kN/m \qquad (14)$$

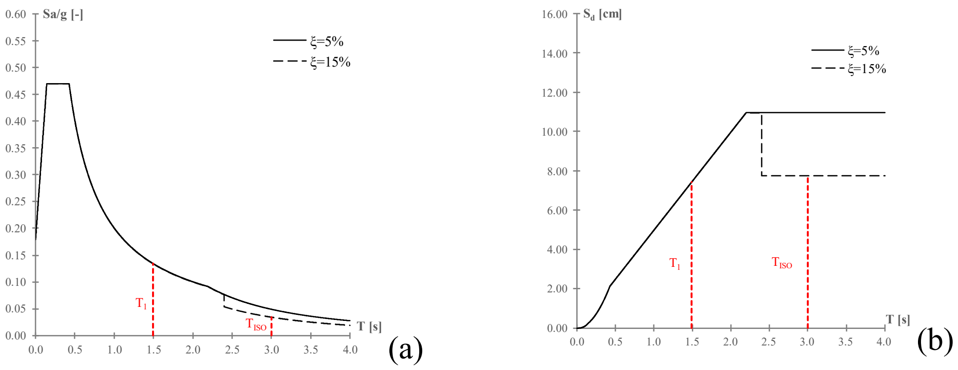


Figure 4 – Elastic spectra at SLC: acceleration (a) and displacement (b)

Assuming two different design solutions that provide the installation of 10 or 20 elastomeric isolators, the horizontal stiffness of each isolator will be equal to 0.81 kN/mm or 0.41 kN/mm, respectively. Table 4 shows the characteristics of the isolators (see Figure 5 for the symbols meaning) for the two different solutions, labelled as “solution 10” and “solution 20”, respectively. Once defined the isolator typology, these were modelled in SAP2000 [13] through two nodes links with a linear behaviour. The vertical and horizontal stiffness of the links are those of the isolator, declared by the producer. The sliding supports, placed on the columns where there are not isolators, are multi-directional confined elastomeric disc structural bearings with a vertical capacity of 1500 kN and displacement capacity of 200 mm. They were also modelled with two nodes links characterized by a vertical stiffness only. The link element allows the relative displacement between the two nodes, simulating the presence of a sliding support.

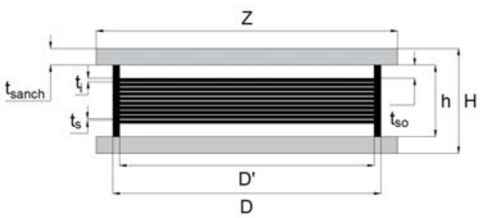
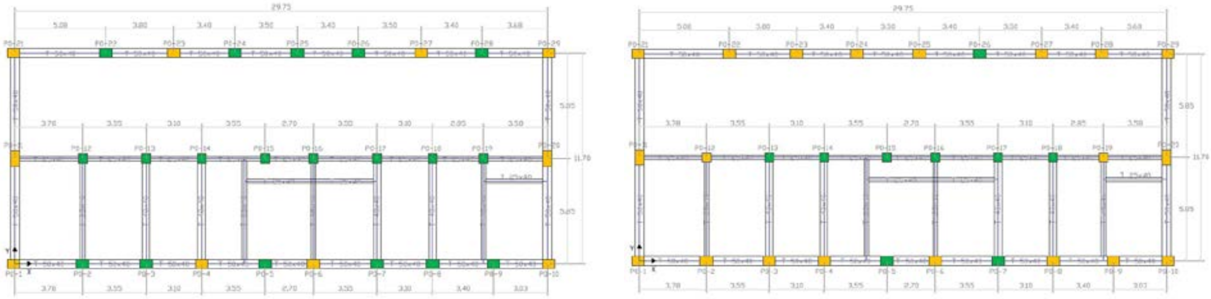


Figure 5. – Seismic isolator (tsanch is the thickness of anchor plates and t_{SO} is the thickness of vulcanized steel plates).

	Solution 10	Solution 20	
V	1150 kN	490 kN	Vertical capacity in seismic conditions (CLS)
$F_{z,Ed}$	4680 kN	2000 kN	Vertical capacity at ULS
K_e	1.01 kN/mm	0.51 kN/mm	Equivalent horizontal stiffness
K_v	1246 kN/mm	519 kN/mm	Vertical stiffness
D	400 mm	350 mm	Diameter of the isolator
t_e	50 mm	75 mm	Total thickness of rubber
h	108 mm	143 mm	Height of the isolator neglecting steel plates
H	158 mm	193 mm	Height of the isolator including steel plates
Z	450 mm	400 mm	Dimension of the anchor plates
W	140 kg	118 kg	Weight of the isolator
d_{cap}	100 mm	150 mm	Displacement capacity
G_{dyn}	0.4 MPa	0.4 MPa	Dynamic shear modulus of rubber (soft compound)

Table 4 – Properties of the seismic isolators

The Figure 6 shows the layout in plan of seismic isolators and sliding supports for the two design hypotheses. The eccentricities between mass centre and effective stiffness centre at the isolation interface is equal to 9 cm in the direction of the long side of the building and 3 cm in the direction of the short side for “solution 10”, while it is equal to 3 cm in both directions for “solution 20”. For both the design hypotheses, 3 modes are sufficient to activate more than 85% of the total mass of the building. The three modes have the following periods: 2.97s, 2.82 s and 1.78s for “solution 10” and 2.96s, 2.81s and 1.61 for “solution 20”, respectively. The isolators are steel reinforced elastomeric ones [14], having a low horizontal stiffness (in order to guarantee the decoupling of the horizontal motion of the structure from the ground one), a very high vertical stiffness (in order to sustain vertical loads without settlements). The geometrical properties of the isolators (global dimensions, thickness of the layers…), as well as the mechanical properties of the elastomer, are the main factors for the definition of vertical and horizontal stiffness. The isolators used in this study are identified through the abbreviations SI-S 400/50 (maximum horizontal displacement of 100 mm), used in “solution 10”, and SI-S 350/75 (maximum horizontal displacement of 150 mm), used in “solution 20”. The elastomeric isolators SI-S 400/50 and SI-S 350/75 have a circular plan shape with diameter equal to 400 and 350 mmm, respectively, and they are realized with soft elastomeric compound (equivalent dynamic shear modulus G_{din} = 0,4 MPa). The total thickness of elastomeric layers, t_e, is equal to 50 mm for SI-S 400/50 and 75 mm for SI-S 350/75, respectively. In Table 5 the geometrical dimensions of the two isolators are shown.



(a) (b)

Figure 6 – Plan disposition of seismic isolators and sliding support for: a) solution 10 and b) solution 20 (isolators in yellow, sliding supports in green)

Table 5 – Geometrical dimensions of the isolators

GEOMETRICAL DIMENSION		**SI-S 400/50**		**SI-S 350/75**	
Diameter	D	400	mm	350	mm
Effective diameter	D’	380	mm	330	mm
Thickness of the single elastomeric layer	t_i	5	mm	5	mm
Number of elastomeric layers	n	10		15	
Thickness of steel layers	t_s	2	mm	2	mm
Thickness of vulcanized steel plates (sup/inf)	t_{so}	20	mm	20	mm
Thickness of anchor plates	t_{sanch}	25	mm	25	mm
Height of the isolator neglecting anchor plates	h	108	mm	143	mm
Total height of the isolator	H	158	mm	193	mm
Dimension of anchor plates (sup/inf)	Z	450	mm	400	mm

Table 6 shows the results of the safety checks of structural elements (beams and columns), that were performed for the Life Safety Limit State (SLV) assuming a behaviour factor q=1.5 (as suggested in [6]). With reference to the maximum ratios D/C it can be observed that: the safety checks of the beams are satisfied for the fundamental combination at ULS and for seismic combinations; the safety checks of the columns are all satisfied for vertical loads combined at the ULS (maximum D/C equal to 0.88 for combined axial load-bending and 0.61 for shear) and for seismic combinations (maximum D/C equal to 0.69 for ductile mechanisms and 0.59 for brittle mechanisms). Table 7 shows the results of the safety checks for seismic isolated building considering the two proposed solutions. For the safety check of the maximum displacement, the capacity values suggested by the producer are considered.

Table 6 – Strengthened structure: safety checks of beams and columns

Check	**Combination**	**Solution 10 $(D/C)_{max}$**	**Solution 20 $(D/C)_{max}$**
Beams	ULS	**0.98**	**1.00**
Bending	Seism	**0.82**	**0.84**
Beams	ULS	**0.69**	**0.68**
Shear	Seism	**0.59**	**0.59**
Columns	ULS	**0.86**	**0.88**
Axial load - Bending	Seism	**0.91**	**0.93**
Columns	ULS	**0.59**	**0.61**
Shear	Seism	**0.57**	**0.59**

Table 7 – Strengthened structure: safety checks of seismic isolators (sec. C.11.9.7 Circ. NTC2018)

Check	Solution 10 $(D/C)_{max}$	Solution 20 $(D/C)_{max}$
Check of the maximum stress in steel plates	0.19	0.26
Check of the total shear strain	0.72	0.86
Check of the buckling load	0.33	0.90
Check of the maximum displacement	0.74	0.49

3.4 Evaluation of the fire vulnerability of the structure

The thermal response of the elastomeric isolators was evaluated with reference to the standard IS0 834 fire curve, using the finite element software SAFIR [15]. In particular, the isolator with a portion of concrete column below and above (70 cm width and 20 cm height), was modelled in order to best reproduce the heat conduction phenomena. Indeed, the portion of the columns below the isolators is bigger, in order to give to the substructure greater strength and stiffness. These reinforced concrete jacketings dimension varied from 70 cm to 105 cm and the most critical case, from the heating transmission point of view, is related to the minimum dimension of column’s side. The thermal properties of steel and concrete are considered variable with temperature, according to EN1991-1-2 [12]. The rubber was modelled as a user defined material, with the following thermal properties: thermal conductivity λ = 0,27 W/Mk; specific heat cp = 1400 J/kgK; density ρ = 1270 kg/m^3. These values were obtained from experimental tests on elastomeric isolators [14] and they are assumed constant with temperature. The entire model (isolator and portions of column) was heated on the lateral side by the standard ISO834 fire curve for 60 min. Figure 7 shows the trend of temperatures during 60 min of fire exposure, in some points placed at half height of each isolator.

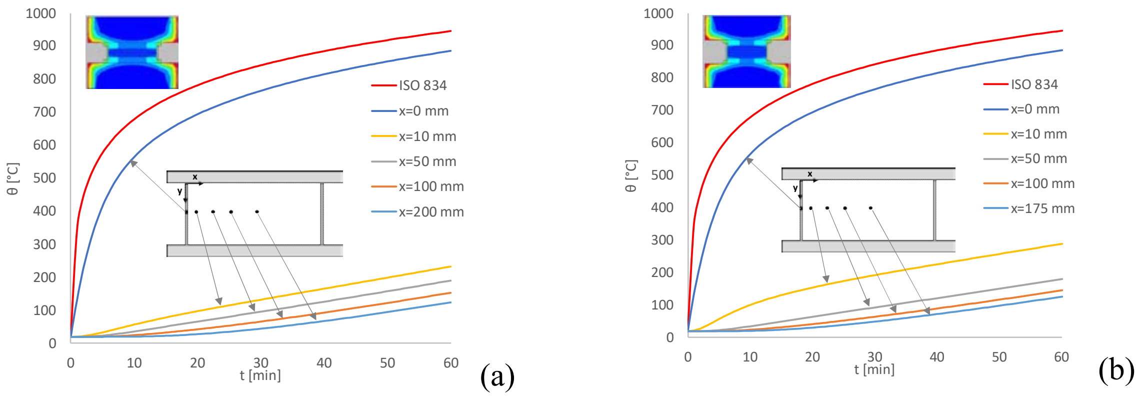


Figure 7 - Temperatures at half isolator height: (a)SI-S 400/50; (b) SI-S 350/75

The outer rubber layer (10 mm thick) that covers the alternating rubber-steel layers acts as a fire protection; indeed, a significant temperature difference between the point at x=0 mm (outer surface of the isolator) and the point at x=10 mm is obtained. The graphs show that in all the points considered the temperature exceeds 100 ºC after 60 min of fire exposure.

Figure 8 shows that the temperatures reached in the middle height of the isolator are the lowest, while moving away from the isolator centre, the temperatures increase due to the presence of the steel end plates. Therefore, relating the isolator damage to the temperature distribution, the damage it is not uniform and it is greater near to the rubber-layers at the steel vulcanized end plates. In this work, the fire assessment of damage isolators is carried out with the standard ISO834 fire curve.

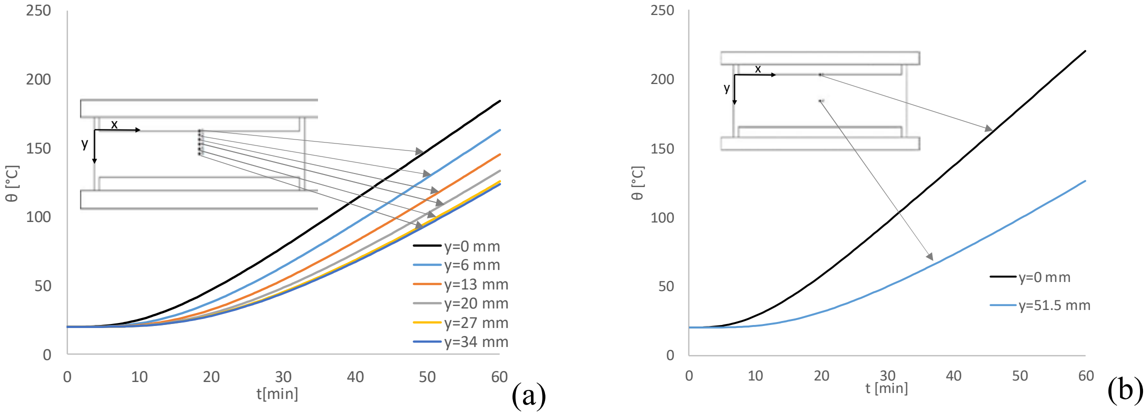


Figure 8 – Temperature evolution along central vertical axis (x = D/2):(a)SI-S 400/50 and (b) SI-S 350/75.

In particular, based on the performed thermal analyses, the thickness of the fire damaged isolator was defined using thermal fields at 30 and 60 min of fire exposure. The damaged zone is defined by the portion of isolator that reaches temperatures above a certain critical temperature threshold (or critical isotherm) at a certain time of fire exposure. In order to associate the isolator damage to the exceeding of the critical temperature threshold, each layer of elastomer has to be investigated, since the temperature distribution in the isolator is not uniform. Therefore, starting from the thermal fields at 30 and 60 min of fire exposure, for each isolator and for each critical temperature threshold, the damaged portion "t_{dam}" of the single layer has been defined as the portion of elastomer characterized by temperatures $\theta>\theta_{crit}$. Figure 9 shows, for the three thresholds of critical temperature (70°C, 200 °C and 320 °C), the reduction factors of horizontal α_{kh} and vertical α_{kv} stiffnesses at 30 and 60 min of exposure to standard fire.

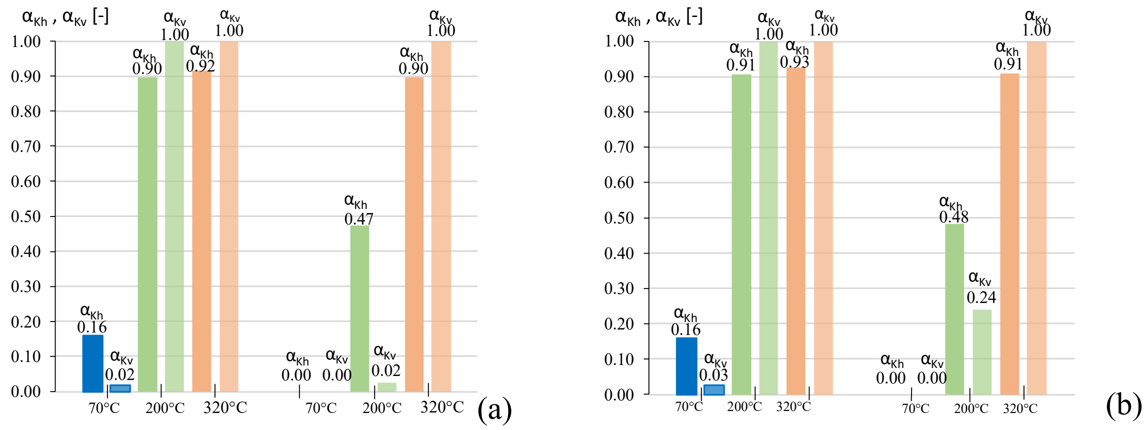


Figure 9 – Reduction factor of horizontal α_{kh} and vertical α_{kv} stiffness for (a) SI-S 350/75 and (b) SI-S 400/50 elastomeric isolators.

Furthermore, Figure 9 shows that the critical temperature equal to 70°C is particularly restrictive, indeed it causes a significant reduction of both horizontal stiffness (-79% for SI-S 400/50 isolator and -84% for SI-S 350/75 isolator) and vertical one (-97% for SI-S 400/50 isolator and -98% for SI-S 350/75 isolator) after only 30 min of exposure to the standard fire curve. In this case, in order to design an adequate passive fire protection system, depending on the fire resistance requirement, the temperature has to be maintained below 70°C. Considering a critical threshold temperature of 320°C, a minimum reduction (-10%) in horizontal stiffness is observed after 60 min of exposure to the standard fire curve for both the isolators, while no reduction of the vertical stiffness is observed, because the temperature threshold is exceeded only in the lateral rubber layer. An intermediate situation occurs with a critical temperature threshold of 200 ºC, in which there is no much degradation of mechanical properties after 30 min of fire exposure. On the contrary, after 60 min the horizontal stiffness of SI-S 350/75 isolator is halved, causing a significant damage of it. Therefore, also in this case, to design a passive fire protection system is necessary, in order to guarantee, after 60 min of fire exposure, that the temperature inside the isolator remains below 200ºC.

3.5 Assessment of load bearing capacity after fire

In order to verify the residual load bearing capacity of the isolators, a comparison with the loads acting on them, derived from the superstructure, was necessary. Therefore, the ULS axial forces acting on the isolators were derived from the SAP2000 model, updating for each case of ($\theta_{crit,\,t}$) the stiffness parameters of the elastomeric isolators damaged by fire; the maximum axial force is considered for the isolator safety check. The tables with the results of the safety checks performed for both the isolators are reported below. The results are expressed in terms of ratio between the demand, D, and the capacity, C. In particular, Table 8 refers to the case in which the full stiffness of sliding supports is considered, while Table 9 contains the results obtained assuming a reduction of the vertical stiffness of sliding supports equal to that assumed for elastomeric isolators. In case of θcrit = 320 °C and θcrit =200 °C, both the isolators don't exhibit load bearing capacity problems, indeed the resistance Fzd is greater than the acting vertical force, NSLU. If the critical temperature is equal to 70°C, after 30 minutes of exposure to the standard fire curve, the isolator SI-S 350/75 is unable to bear the vertical loads from the structure, requiring fire protection.

Table 8 – SLU load-bearing capacity safety check

		Sliding support with stiffness degradation						**Sliding support without stiffness degradation**					
		SI-S 400/50			**SI-S 350/75**			**SI-S 400/50**			**SI-S 350/75**		
θ_{crit}	t	F_{zd} (C)	N_{SLU} (D)	D/C	F_{zd} (C)	N_{SLU} (D)	D/C	F_{zd} (C)	N_{SLU} (D)	D/C	F_{zd} (C)	N_{SLU} (D)	D/C
[°C]	[min]	[KN]	[KN]	[-]	[KN]	[KN]	[-]	[KN]	[KN]	[-]	KN]	[KN]	[-]
70	0	4680	1297	0.28	2000	1332	0.67	4680	1297	0.28	2000	1332	0.67
	30	1091	574	0.53	359	849	2.37	1091	1193	1.09	359	1174	3.27
	60	-	-	-	-	-	-	-	-	-	-	-	-
200	0	4680	1297	0.28	2000	1332	0.67	4680	1297	0.28	2000	1332	0.67
	30	4680	1297	0.28	2000	1332	0.67	4680	1297	0.28	2000	1332	0.67
	60	2505	1162	0.46	1063	909	0.86	2505	1279	0.51	1063	1170	1.10
320	0	4680	1297	0.28	2000	1332	0.67	4680	1297	0.28	2000	1332	0.67
	30	4680	1297	0.28	2000	1332	0.67	4680	1297	0.28	2000	1332	0.67
	60	4680	1297	0.28	2000	1332	0.67	4680	1297	0.28	2000	1332	0.67

3.6 Fire protection system for seismic isolators

The assessment of the fire effects on seismic protection systems is a relevant topic for different aspects and problems. For example, a localised fire involving a limited number of elastomeric isolators, may causes problems for vertical loads due to differential displacements of the isolation system generated by vertical stiffness degradation. In addition, the horizontal stiffness degradation can lead to not satisfying the designing assumption in which the centre of mass and stiffness must be, as far as possible, coincident. Furthermore, a localized degradation could cause a displacement of the stiffness centre, generating an eccentricity that can cause additional torsional effects on structural elements. Since it is difficult to experimentally characterize the thermo-mechanical properties of the seismic isolators, the conservative assumption of θcrit =70°C was adopted in this work and a fire protection system of the isolators was considered to reduce the fire induced damages. In particular, a protective box of 25 mm thick gypsum plasterboards with a variable thermal conductivity (see Table 10) was considered (see Figure 10).

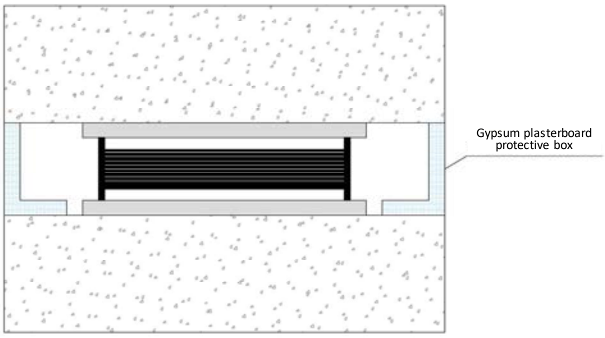


Figure 10 – Protective box installation.

Table 9 – thermal properties of gypsum plasterboard

	λ	c	ρ
[°C]	[W/MK]	[J/kg°C]	[kg/m^3]
20	0.20	800	800
65	0.80	800	800
70	0.80	5000	800
100	0.35	5000	800
102	0.35	5500	800
126	0.35	10000	800
140	0.12	10000	800
150	0.12	3050	800
370	0.12	3050	800
600	0.20	3050	800
1200	0.20	3050	800

The designed fire protection system ensures that the critical temperature of 70 ºC on the outer surface of the isolator is not exceeded after 60 min of fire exposure to the standard fire curve. Therefore, the isolators do not suffer any reduction in mechanical properties and the isolation system maintains its effectiveness

4 CONCLUSIONS

The paper shows the methodology to assess the seismic vulnerability after fire of buildings seismically isolated at the base. The methodology was applied to an existing multi-storey reinforced concrete building by considering two retrofit interventions with seismic base isolation. The thermal field in seismic isolators after 30 and 60 minutes of exposure to the standard fire curve ISO834 was assessed to estimate the degradation of vertical and horizontal stiffness and load bearing capacity of the isolators. This degradation was estimated based on the thermomechanical properties of the elastomer layer damaged by fire, considering the portion of isolator that achieve temperatures higher than a certain threshold (70, 200 or 320 ºC).

The results of the analyses show that:

- the experimental characterization of the behaviour of the elastomer at high temperatures would allow to evaluate with greater accuracy the need for a passive fire protection system, that should guarantee, according to the fire resistance requirement, that the critical temperature threshold θcrit is not exceeded;
- for both the cases θ_{crit} = 320 °C and θ_{crit} = 200 °C, the isolators do not show any problem in terms of load-bearing capacity;
- the case θ_{crit} = 70 °C is the most severe, since after 30 min of exposure to fire the isolator SI-S 350/75 is not able to sustain the vertical loads due to the superstructure, and requires a fire protection system.
- a passive fire protection system consisting of a protective box made of plasterboards panels, with an “L” section, guarantees that the critical temperature of 70 ºC is not exceeded on the external surface of the isolator after 60 minutes of exposure to the standard ISO834 fire curve and, therefore, that isolators do not suffer any reduction of mechanical properties.

The stiffness degradation is uniform due to the assumption of generalized fire. Is likely that localized degradation in case of localized fire change the position of the stiffness centre and this can cause additional torsional effects, which we did not consider. Moreover, in order to evaluate the static and seismic behaviour after fire of base isolated structures, in which the isolation system typically consists of both isolators and sliding supports, also the thermal degradation of the sliding supports must be considered. The ratios between the vertical loads on isolators and sliding supports before and after fire, varying the critical temperature threshold and taking into account or not the degradation of the sliding supports, show that:

- the vertical loads on the isolators reduce and consequently those in the sliding supports increase;
- the redistribution of the vertical loads on isolators and sliding supports is greater when the stiffness degradation of the sliding supports is not considered;
- assuming an equal degradation of all the seismic isolators, the more numerous are the isolators with respect to the sliding supports, the more the redistribution of vertical loads is high;
- the solution with more sliding supports is more efficient after a fire event.

The approach herein shown could be extended, in future, to fire scenarios with temperature curves different from the nominal standard curve. Moreover, localized fire scenarios could be considered, in order to take into account the effects of differential displacements that may occur when the degradation of seismic isolation devices is not homogeneous.

REFERENCES

[1] Yasser M. Al-Anany, Michael J. Tai (2017) - Fiber reinforced elastomeric isolators for the seismic isolation of bridges – Composite Structures 160 (2017) 300-311 https://doi.org/10.1016/j.compstruct.2016.10.008

[2] Traxl R, Maier M, Bauer S, Schaur A, Zeiml M, Lackner R. (2018) - Thermochemical assessment of the load-bearing capacity of steel-reinforced elastomeric bearings subjected to fire loading - Engineering Structures 160 (2018) 12-23

[3] Fulin Z, Bo W, Ping T, Chaoyong S, Liwei H, Wu B. (2011) - Experimental study on fire resistance of building seismic rubber bearings – J. Struct. Eng. (2011) 137(12): 1593-1602

[4] Mazza F. (2017) – Residual seismic load capacity of fire-damaged rubber bearings of reinforced concrete base-isolated buildings- Engineering Failure Analysis 79 (2017) 951-970

[5] EN1992-1-2 (2004): Eurocode 2: design of concrete structures, part 1-2: general rules – structural fire design, Brussels (Belgium).

[6] D.M.II.TT. 17-01-2018: “Aggiornamento delle Norme tecniche per le costruzioni” (G.U. n. 42 del 20 febbraio 2018). (in italian)

[7] Circolare Ministero delle Infrastrutture e dei Trasporti del 21/01/2019 n. 7: Istruzioni per l’applicazione dell’aggiornamento delle “Norme Tecniche per le Costruzioni” di cui al D.M. 17/01/2018. (in italian)

[8] EN 1998-1 (2004): Eurocode 8: Design of structures for earthquake resistance – Part 1: General rules, seismic actions and rules for buildings [Authority: The European Union Per Regulation 305/2011, Directive 98/34/EC, Directive 2004/18/EC]

[9] ISO 188 Rubber, vulcanized or thermoplastic — Accelerated ageing and heat resistance tests, Int. Stand. (2007) 13.

[10] Losanno, D., Palumbo, F., Calabrese, A., Barrasso, T., Vaiana, N., Preliminary Investigation of Aging Effects on Recycled Rubber Fiber Reinforced Bearings (RR-FRBs), Journal of Earthquake Engineering, 2022, 26(10), pp. 5407–5424, DOI 10.1080/13632469.2021.1871683

[11] Trombetta P.L, Castellano M.G, Cocchio D. (2013) - Adeguamento Sismico di Edifici mediante Isolamento Sismico – In Atti del XV Convegno ANIDIS (Associazione Nazionale Italiana di Ingegneria Sismica), Padova (PD), 30 June / 4 July 2013 (in italian)

[12] EN 1991-1-2, 2002: EUROCODE 1: "Action on structures - Part 1-2: General actions- Actions on structures exposed to fire"

[13] SAP2000 (Version Ultimate 21.0.2) – Computers and Structures, Inc.

[14] Catalogo FIP INDUSTRIALE- Leading technologies. Isolatori elastomerici serie S1 (in italian)

[15] FRANSSEN J.M., "User's Manual for SAFIR2016c – a computer program for analysis of structures subjected to fire." University of Liege, Belgium 2016.